SIGN STRUCTURES GUIDE

SUPPORT DESIGN FOR
PERMANENT
UK TRAFFIC SIGNS
Revised for Eurocodes and passive safety

September 2010
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Support design for permanent UK traffic signs to BS EN 12899-1:2007 and structural Eurocodes

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2010 Revision
In this edition passive safety has been given greater coverage and all references and calculations have been updated to accord with the latest standards and their UK national annexes. The style and language used more closely matches that of the Eurocodes, although the authors have retained the UK decimal point. Foundation design is now limit state and in accordance with Eurocode 7. Guidance has been added on wind funnelling.
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1. INTRODUCTION

1.1 The design of supports for traffic signs has changed significantly as a result of the introduction of European standards. This booklet attempts to clarify the current situation and to provide a reference for all those involved with traffic signing. Much of Appendix C requires some understanding of structural engineering, but the bulk of this publication is intended for use by traffic and highway engineers: the people who most often need to specify sign structures. In conjunction with suitable computer software, it will cover most situations. It is nevertheless recommended that the advice of a structural engineer should be sought in cases of any doubt and always for larger signs and those mounted above the carriageway.

1.2 For many years sign structure design in the UK was standardised, a single wind pressure being used throughout the country. However, the relevant standard, BS 873, was withdrawn at the end of 2005, being replaced initially by BS EN 12899-1:2001 and now by BS EN 12899-1:2007. One of the changes from BS 873 is the need to specify what wind pressure each sign needs to withstand. There is thus an additional step in the design process that this Guide helps to explain.

1.3 The European standard offers a wide range of classes for each aspect of a sign’s performance, in order to address the requirements of all participating countries. To assist designers of traffic signs in the UK and to encourage consistency where it is appropriate, the UK version of the standard, BS EN 12899-1:2007, has a National Annex appended to it. This contains recommended values and classes for the options, including for wind loading, as is therefore an invaluable reference.

1.4 The determination of the wind load and the design of the structure are two essentially separate tasks that do not necessarily need to be undertaken by the same person or organisation. It is recommended that when a wind load (or wind action to use the terminology of the Eurocodes) is recorded or communicated between organisations as part of a sign specification, this should be the basic wind pressure, $w_b$. This is the load prior to the application of any safety factor or force coefficient, and is equivalent to and comparable with the values in Table 8 of BS EN 12899-1:2007 and table NA.2 of its National Annex. As the nine WL classes in BS EN 12899-1:2007 differ from those in the previous edition, the use of the basic wind pressure is recommended in preference to stating a WL class to avoid any confusion as to which version of the standard is intended, and to permit the range to be extended and intermediate values specified.

1.5 The UK National Annex to BS EN 12899-1:2007 includes a table of suitable wind pressures, broken down by country or region and overall sign height. It does not address signs over 7 m total height, nor those at an altitude of more than 250 m above sea level. This advice is the result of work commissioned by the Highways Agency and significant discussion amongst industry experts. The intention was to produce guidance that was (a) reasonably simple to use, (b) complied with current standards on wind actions and (c) resulted in economic support sections that were comparable with those typically specified under previous standards. Inevitably this requires a much more detailed appraisal of the sign and its location than using a blanket wind load value. This booklet explains how to use this new method.

1.6 Those involved in designing the faces of traffic signs, particularly directional signs, should be aware that they frequently have a choice of layout and that the size and cost of the supporting structure will depend upon how they use this freedom. For example, a tall narrow sign requires stronger posts and foundations than a ‘landscape format’ sign of the same area. Even where the available width is constrained, preventing any major rearrangement of the layout, it is often possible to reduce the sign face area by altering the positions of text and symbols to eliminate some of the blank space. In certain situations splitting directional information across two separate signs in advance of a junction is preferable to using a single large sign, both for driver perception and structural reasons. However, when
attempting to reduce the size of a sign, on no account should the size of lettering or the layout guidance in the Traffic Signs Manual and DfT working drawings be compromised, as this would degrade the readability of the sign and make it less capable of fulfilling its purpose of helping the road user.

**Sign structure design**

1.7 Sizing the supports for a sign is a matter of balancing risk and economy. The most obvious risk is that a traffic sign structure might fail in strong winds, leading to the economic cost of replacing it and the safety and other disbenefits of the sign being absent in the meantime. There is also the danger that a failed or failing sign structure might cause damage or injury as it falls or as a result of it obstructing the highway.

1.8 However, the risk associated with the failure of a properly designed sign structure is negligible compared to the risk that a vehicle might leave the carriageway and collide with it. For this reason passively safe (or crash friendly) supports are increasingly specified for use on classified or well-used roads. A now superseded Highways Agency standard required safety fence protection or passively-safe supports on roads with a 50 mph or higher speed limit, but the National Annex to BS EN 12767:2007 now recommends that passive safety should be a consideration on all roads. Passively safe signs must still meet the structural requirements of BS EN 12899-1, although a greater deflection is permitted. Passive safety is considered in more detail in Section 2. There is always a safety benefit as well as an economic one in using the most slender support that is structurally adequate.

1.9 Over-sized supports cost more, add to visual intrusion and are more likely to lead to death or injury than correctly designed ones. Over-design may also result in unnecessarily large foundations, leading to a longer construction period and thus greater disruption, coupled with a greater difficulty of locating such bases in urban roads containing extensive services. It is therefore well worth the trouble of designing sign structures using the method described in this publication, which aims to minimise support sizes whilst adhering to the relevant standards and sound engineering practice.

**Wind actions**

1.10 The most significant force on a sign is that exerted by the wind. Determining wind actions involves an element of probability and some knowledge of the physical characteristics and topography of the location concerned. EN 12899 uses the term *wind load*, which is synonymous with *wind action*.

1.11 The relevant standard for wind actions is BS EN 1991-1-4:2005, which is referenced by BS EN 12899-1. The UK National Annex to BS EN 1991-1-4 includes a map of the UK showing the base reference wind speed for any location. To this value a number of corrections need to be applied to allow, for example, for the altitude, the height and ground clearance of the structure, its proximity to the coast or large inland lake and whether it is within a town or rural situation. Consideration also needs to be given to any local funnelling effect that might occur between tall buildings in a town environment, in a valley with a significant narrowing, or whether there is a sharp rise in ground level close to the relevant location. It is thus time consuming and generally uneconomic to derive an accurate wind load assessment individually for each sign location.

1.12 The National Annex to BS EN 12899-1:2007 recommends suitable wind loads for the majority of signs in the UK. Whilst is it more conservative than performing a full analysis, it is simpler and quicker. The method is not suitable for all situations: it cannot be used at an altitude of greater than 250 m, or where there is significant wind funnelling or topographical features such as cliffs or scarps that affect the wind flow. Locations with these effects are referred to in the Eurocode as having significant *orography*, and the recommended action is to revert to BS EN 1991-1-4. This Guide consequently gives examples of both methods of calculation.
1.13 At present there is no published guidance available for design in areas of wind funnelling. A study is currently being carried out to define where funnelling occurs and to determine simple procedures to be adopted. Section 6 contains interim guidance on this subject.

1.14 The 12899 National Annex suggests that authorities responsible for signs within a defined area may wish to derive a wind load that can be applied to all their signs, or to sub-divide their area into regions with similar wind characteristics. This will result in significant economy for areas located towards the south or east of their country or region or where the maximum altitude of any public highway is well below the 250 m assumed in the National Annex. It will also make subsequent design work easier for authorities that have areas for which the simplified method cannot be used. The work involved in applying the methodology of BS EN 1991-1-4 a single time will be amply repaid for every sign that is subsequently designed to the wind load thus obtained.

1.15 In practice, most designers of sign structures will use appropriate computer software. They nevertheless need an overview of the methodology involved to ensure that they are using the software correctly, with suitable options and parameters selected, and to check that the results obtained are reasonable. A reference to software available free of charge is given in Appendix D.

Other actions on signs

1.16 Other forces that may need to be taken into account when designing a sign structure are point loads and dynamic snow load. The UK National Annex recommends that signs should be able to withstand a force of 500 N applied at any point. This represents the load that might be exerted by, for example, a glancing blow from a vehicle mirror, a falling branch or malicious interference with the sign. This point load is the critical factor only for very small signs, but for signs mounted on a single support it causes torsional forces that need to be considered.

1.17 Dynamic snow load is relevant only to low-mounted signs in areas where snow ploughs or snow blowers are regularly used if there is a significant problem of sign damage caused by these operations. The BS EN 12899-1 National Annex recommends that class DSL1 (1500 N/m²) is relevant to snow blowing and class DSL2 (2500 N/m²) to snow ploughing at up to 40 mph. As snow clearance is likely to be regular only in areas of high wind, and the dynamic snow load is not applied to any portion of the sign higher than 2.5 m from the ground, it will be unusual for the snow load to be the critical factor in sign structure design. It is therefore rare for snow loading calculations to be required.

Comparison with previous methodologies

1.18 Prior to the introduction of BS EN 12899, the structures for most small and medium-sized traffic signs were designed using the DfT nomograms or a computer simulation of them. These charts with the reference WBM140 were first produced for the 1967 edition of Chapter 13 of the original Traffic Signs Manual and were last updated in 1983. They continued to be used, despite being based upon superseded British Standards, as there was no better guidance readily available.

1.19 The nomograms provided a choice of wind pressures and Chapter 13 recommended 1530 N/m² for signs in exposed positions and 960 N/m² for signs in urban areas and sheltered places. This pressure was rounded to 1500 N/m² in later revisions of the former traffic signs standard, BS 873. A comparison between the methodology of BS 873 and the BS EN 12899-1 National Annex shows that broadly similar sections are achieved in England, but that in Scotland a larger section than would have been used previously is generally indicated. The difference for signs on a single post is particularly marked because the nomograms used a more onerous deflection criterion for signs on more than one support.

1.20 BS EN 12899-1 and BS EN 1991-1-4 require the use of more conservative force coefficients and partial action (safety) factors than BS 873 or the nomograms. Hence, for a typical small sign, the BS 873 standard wind load of 1500 N/m² results in a broadly similar size of support section to a load of 1100 N/m² using the current method of calculation.
2. PASSIVE SAFETY

Introduction

2.1 Traffic signs, lighting columns, and other highway structures provide a useful function in making the highway safe and usable and providing driver information. However, designers should be aware that objects installed by the roadside pose a risk of injury to the occupants of any vehicle that leaves the carriageway. Passively safe structures are designed to provide less resistance during impact reducing the severity of that impact for the occupants. The performance levels are confirmed by testing.

2.2 All structures need to comply with the structural requirements, which for traffic signs is BS EN 12899-1. Passive safety is an additional characteristic – it does not override the need to ensure that the structure can support the required loading. The only relaxation is in the deflection requirements detailed in BS EN 12899-1 NA.2 (extract included in Appendix A).

Why use passively safe posts?

2.3 The photographs below indicate the different levels of damage incurred in similar impacts. The traditional post would have resulted in probable death for vehicle occupants whereas it is unlikely that any injuries would have been incurred hitting the passively-safe post.

![Traditional post](114mm wide base) 
vehicle badly damaged, rapid deceleration

![Passively safe post](140mm dia) 
minor damage to bonnet/bumper 
vehicle still driveable, low deceleration.

Costs

2.4 Passively safe posts are generally more expensive than the equivalent steel post. This may influence the decision to specify them, but this must be balanced against the significant reduction in risk. Other factors may have to be considered. The omission of vehicle restraint systems can reduce scheme costs. Electrical disconnection systems (where required) can add costs but they allow rapid replacement of the post if struck, reducing the disruption and costs. This may be an issue at certain sites. As with every scheme, this is a balance between available budget and the benefits of the various risk reduction measures.

Where to use Passively Safe Structures

2.5 Passively safe structures have been used predominantly on high-speed roads where the benefits are significant. In urban situations where speeds are lower they still have benefits, but the presence of obstructions such as buildings and parked cars, and anything that restricts vehicle speeds, such as traffic calming and road geometry, will reduce the benefits. In these situations a collision with the structure in question at a speed sufficient to cause it to behave passively becomes less likely, and there may be insufficient room for the structure to deflect if adjacent to a wall or building.
2.6  Passively safe products are part of the wide armoury of tools available to improve road safety. Designers need to be aware that they will not be the best solution in all circumstances. Limitations on budgets restrict possible solutions, so designers will need to assess the optimum use of available funding. Simple measures like renewing white lines or using anti-skid treatment may be preferable where larger areas can be treated for the same budget. But when used effectively to reduce the risks at specific locations, identified by accident records or risk assessment, passively safe structures allow the benefits of having the sign in the desired location whilst reducing the risk associated with a traditional post.

Risk Assessment – Individual Structure

2.7  When assessing requirements for any site it is recommended that designers compile a number of queries that need to be addressed. Some typical questions/procedural steps are listed below:

1. Is the structure really needed? Is it required for safety reasons or to comply with legislation or best practice, or has the safety audit identified this as necessary?
2. Does it have to be that size and weight? Directional signs can often be made smaller (without reducing the size of lettering) with careful layout changes.
3. If it is essential, can it be relocated to a safer position?
4. If not, consider a passively safe structure.
5. If there are other hazards, like bridge piers, that need protecting with a Vehicle Restraint System (VRS) use traditional sign supports.
6. Where it is proposed to install a sign behind a VRS protecting an existing structure and it is likely that an existing structure will be removed in the near future, negating the need for VRS. Then consideration should be given to installing a passively safe structure, as this will allow the VRS to be removed without any additional need for protection of new structures.

2.8  Risk assessment is the principle that must underpin all projects. Designers are required to demonstrate that they had a robust assessment method for determining risks associated with each scheme, that incorporates a methodology for evaluating improvements to existing hazards against associated costs of incorporating this within the scheme. Designers should commence with the objective of minimising all street furniture. If there are no obstructions on the verge then the risks to the driver of an errant vehicle are significantly reduced. The use of passively safe structures normally allows installation without the need for VRS, which is of course itself an obstruction. So along a typical length of carriageway there will be fewer objects to hit and the probability of a collision will be reduced. Individually passively safe structures are more expensive than traditional steel posts however the omission of VRS can realise significant cost savings on most schemes in addition to reducing risks.

Primary Risk

2.9  This is the first of two main considerations. When a vehicle strikes a structure, such as a lighting column, at moderate to high speed, it is probable that the car will suffer serious structural damage. During a high-speed impact, even if the car remains relatively undamaged, the forces on the internal organs of vehicle occupants can cause fatal or serious injuries, despite the use of seat belts and air bags. By altering the properties of the column, the consequence of a vehicle strike can be greatly reduced. This is the principle behind passive safety and, correctly applied, does significantly reduce the risks to the vehicle occupants.
Secondary accidents

2.10 The second consideration is the possibility that, after the initial impact, the vehicle will continue unrestrained leading to a further accident, or that the debris will cause injury to other road users.

2.11 These two risks cannot be considered equal. A study of the behaviour of structures during crash tests indicates that the ‘debris’ will generally fall back over the vehicle at high speed and forward at low speed, and in either case be deposited close to its original position. Therefore, where a structure is situated on the verge the probability is that debris will remain on that verge. For a secondary accident to occur the post has to be struck and fall, or be dragged onto the carriageway, and then be of sufficient size to cause damage or an accident. This also assumes that the oncoming vehicle cannot stop or avoid the debris. Risk assessments should recognise that primary and secondary accidents are a different order of probability: secondary accidents are very rare occurrences and therefore a very low risk.

Risk Assessment – Global environment

2.12 It is important that designers consider the whole roadside, not just an individual structure. This is particularly important in small projects where only one or two structures will be replaced, but other existing structures will remain. The full benefits of expenditure on passively safe structures will only be realised if all the risks associated with the location are considered and mitigated. Drivers make mistakes and the ultimate aim for all designers is to provide a roadside that is more tolerant and forgiving.

2.13 All structures pose a risk to drivers during impact. Traditional structures present a much greater risk than passively safe structures. If designers install passively safe structures on a roadside without considering the adjacent structures they should be aware that their proposals could increase the risks associated with that stretch of road if they have simply added additional potential hazards without addressing the underlying problem. This may be difficult to justify in a road safety audit.

2.14 Installing a new passively safe sign close to, say, a traditional unprotected lighting column undermines the principles of EN12767. Every attempt should be made to address existing hazards within any new scheme. A major cost on most schemes is traffic management, for a minimal additional cost some hazards could be removed. Designers should seriously consider the risks associated with leaving existing hazards untouched. It is the designers’ responsibility to assess the risks associated with each proposed scheme and devise a solution that best addresses those risks. It is recommended that all decisions are recorded especially where existing hazards are not being addressed.

Performance Classes

2.15 UK engineers are increasingly specifying passively safe products. However it is not sufficient to specify any product without having an appreciation of the differences between product classifications, and the impact speed for which they have been designed and tested. For structures to be declared passively safe they must be crash tested in accordance with EN 12767. The results of these tests will determine the classification of the post. There are three main categories of passively safe structure:

- high energy absorbing (HE);
- low energy absorbing (LE);
- non-energy absorbing (NE).

EN 12767 describes the distinction between the energy absorbing and NE classes thus:

Energy absorbing support structures slow the vehicle considerably and thus the risk of secondary accidents with structures, trees, pedestrians and other road users can be reduced.

Non-energy absorbing support structures permit the vehicle to continue after the impact with a limited reduction in speed. Non-energy absorbing support structures may provide a lower primary injury risk than energy absorbing support structures.
Posts that are tested and do not comply with the requirements or are not tested can be classified as Class 0 to EN12767, but Class 0 posts are not passively safe.

Typical failure modes to be expected for different energy absorption classes

2.16 Non-Energy absorbing structures (NE) provide the lowest risk to the vehicle occupants, but the vehicle will travel further and may hit another object. With High Energy absorbing posts (HE) the vehicle will be decelerated to under half its initial speed, but the risk to its occupants increases. There is no one solution suitable for every scheme, but passively safe supports for traffic signs are overwhelmingly of the NE class.

2.17 The speed value (100, 70 or 50), placed before the energy absorption class, indicates the maximum speed (km/h) at which the product has been tested, and should accord with or exceed the known or measured 85th percentile speed of traffic on the road (not necessarily the speed limit). All products have also been tested at 35 km/h. A third class, the occupant safety level, appears after the energy absorption class, but its value may be specified as 1-3 (i.e. any class acceptable) for UK use.

2.18 The most common specification of EN 12767 classes for a traffic sign support in UK is:

100:NE:1-3

which means a support tested at 100 km/h that is non-energy absorbing and has any occupant safety level.

Practical use of passively safe products

2.19 An understanding of the performance characteristics of different passively safe products can assist designers, but some caution is needed to ensure that an appropriate solution is proposed.

2.20 At locations with a high density of non-motorised users (NMUs) some designers have proposed High Energy posts in the belief that these would slow the vehicle and prevent secondary accidents. What must be remembered is that it is not a normal function of a traffic sign to restrain errant vehicles.
Were a sign no longer to be needed, its supports would not be retained just for their restraining effect, and a sign is unlikely to be in the right location to protect a NMU. Therefore, alternative protection should be considered if there is a significant risk to NMUs. The test procedure for EN12767 involves recording the exit speed at a distance beyond the structure: for a 12 m lighting column this would be 12 m in the direction of travel. Even at that position, a 100:HE post would have an exit speed of up to 50 km/h, which might still present a significant risk to NMUs. The designer therefore needs to balance the risk to vehicle occupants in achieving the greatest reduction in speed against those to NMUs in not sufficiently restraining errant vehicles.

2.21 Urban roads with a high NMU density generally have speed restrictions, so the risks are reduced. Other considerations are roads where the density of daytime traffic limits maximum speed, and conversely that many single vehicle collisions with roadside objects occur at night when fewer NMUs are about.

2.22 Locations such as nosings, slip roads, roundabouts and central reserves need careful consideration. Safety barriers may not be suitable where, for example, the impact angle would be too steep or the barrier could block visibility. A passively safe support without further protection is often a satisfactory design solution. The slight increase in the already small risk of secondary accidents is outweighed by the significant benefit of being able to install a sign or signal at the optimum location. Risk assessments for traffic signal installations justify the use of passively safe posts for most locations on this basis.

2.23 For high-speed dual carriageways and motorways it is likely that an existing vehicle restraint system will already be in place, and it is important to take into account that they deflect when struck. The working width is the zone from the traffic face of the barrier, before impact, to the extreme position of any part of that barrier on impact. This varies depending on the specific VRS (see table 4 of EN1317-2). If a structure were installed within this working width it would restrict the deflection of the barrier during impact, significantly increasing the risk to the driver. Also there would be a risk of more damage to the vehicle and of a secondary accident, as the vehicle might not be deflected back onto the carriageway but put into an uncontrolled spin.

2.24 Serious reflection should be given to any suggestion that proposes installing a structure close to the barrier. Designers must justify the need for that structure at that particular location. In exceptional circumstances consideration may be given to small structures that would provide minimal resistance during impact. Some guidance is given in TD 19/06: Requirements for Road Restraint Systems (DMRB Volume 2, Section 2, Part 8).

Certification

2.25 Designers must satisfy themselves that the product selected meets the requirements of their design. This may be achieved by certification, third party checks or specific checks to the client requirements.

2.26 Where products carry a CE mark, the properties of the product will have been verified by a Notified Body and can be accepted without further checks, provided the classes to which the product conforms accord with those specified.

2.27 Without CE marking designers have to examine the test reports to compare the results with the standard and determine acceptability for every proposed product. To ensure compliance with EN12767, a designer requires an understanding of the test procedures and the way the results are reported. Many test reports do not provide a summary and the designer needs to be able to interpret the results to determine the suitability of any particular product.
2.28 In order to avoid this necessity, a system of ‘third party checks’ has been used by many manufacturers and accepted by client organisations. To achieve this, an independent body or person, such as a consulting engineer, examines the documentation and verifies compliance.

Steel posts

2.29 The Highways Agency carried out testing to demonstrate that small steel posts are passively safe. This work has been incorporated into BS EN 12767. These are an exception to the requirement that individual products should be tested, as generic approval (deemed to comply) has been given irrespective of manufacturer. Clauses 5.4, NA 5.1 and Annex F record that posts of 89 mm diameter and 3.2 mm wall thickness or smaller (of steel grade S355J2H or lower) are passively safe, and 76 mm diameter posts can be used at 300 mm centres (or 750 mm centres if three or more supports are needed). The Annex does not specify precisely which rectangular steel supports or circular sections smaller than 89 mm are deemed passively safe, but most authorities are happy to accept the former Highways Agency advice that any support with a lower moment capacity than an 89 mm $\times$ 3.2 mm S355 grade CHS is acceptable. Many standard aluminium sections are also considered passively safe for similar reasons.

2.30 Standard sections can be a cost effective alternative to products specifically marketed as passively safe. They are would generally only be suitable for smaller signs but in an urban location this would cover many of those used. Sufficient capacity for their use can frequently be obtained by using the more detailed and site-specific wind loading calculation method of BS EN 1991-1-4, described in this Guide

Interpreting the standard

2.31 EN 12767 is specific in the requirement that a product is tested for the intended use. A post tested as a lighting column cannot be assumed to be passively safe if it supports a speed camera. If a product was tested with a thin aluminium sign plate it must not be assumed to be acceptable for supporting a variable message sign (VMS). The structural capacity of the post may be adequate for other applications, but without the test data it cannot be marketed as passively safe.

2.32 However, risk assessment may be utilised to consider using products in demonstrably similar or less onerous circumstances not fully covered by available test data. For example, a post tested supporting a very large sign could be considered suitable to support a small CCTV camera. There are no guidelines that can be given as an acceptance criterion, so the designer has to justify these by demonstrating that the benefits are considerably greater than the risks.

2.33 For sign supports it is normal to test the structure with a sign mounting height of 2 m, although a number of proprietary supports have now been tested satisfactorily at 1.5 m. This height of 1.5 m is common in UK, as it accords with DfT recommendations for rural areas, and a higher mounting height would require larger supports increasing the risks. On most cars the windshield is generally between 2 m and 1.5 m. The lower the proposed mounting height, the greater the risk to the driver that the sign or its supports will puncture the windshield. Designers have to seriously consider the risks associated with specifying a lower mounting height than the proposed product has been tested to. For example, a VMS or other heavy sign at a low mounting height carries a considerable risk of puncturing a windshield on impact, whereas a light gauge aluminium plate or composite material with small channels has an acceptable risk.

2.34 The spacing of two or more posts must usually provide a clear opening of 1.5 m at the impact angle of 20° unless the supports have been tested specifically for impact on more than one post, or standard steel sections are being used. Enhanced aluminium channel sections are available to stiffen the sign face allowing a wider than usual span between supports or a wider overhang either side of a central post. However, confirmation should be sought from the supplier that this will not affect the performance during impact. Use of such sign plates helps to achieve the necessary spacing, but
positioning passively safe posts remains a challenge, particularly where there is a footway or cycleway than cannot be obstructed.

2.35 Where a sign in a cutting has multiple posts, designers have queried whether the measurement of mounting height should be taken at the nearside post or the one furthest from the carriageway. The back post is further from the carriageway, so the probability of impact is less. Many designers have rationalised that it may be acceptable to consider a slight increase in the risk of a lower mounting height at the back post, as this can be balanced against the reduced chance of an impact.

2.36 Making the road environment more forgiving is an aspiration for all designers and will have significant safety benefits. The use of passively safe structures can be an effective solution to reducing the risks associated with vehicle impact. These structures have different requirements to traditional steel posts, which designers will need to understand and take account of. This may require appreciation of the general principles, specific knowledge of the product as well as a global perspective on all the available options.

2.37 Road safety is an important issue with a high public profile. Government targets are challenging. The available budgets are being stretched so designers have to constantly review and adapt possible solutions to maximise benefits. Passively safe structures can provide a cost effective solution whilst minimising risks. It is recommended that designers familiarise themselves with the basic principles to allow more effective consideration of improvement options.

Further information

2.38 The publication “Passive Safety UK Guidelines for Specification and Use of Passively Safe Street Furniture on the UK Road Network” provides guidance for designers. The Passive Safety UK website (see Appendix D) has this available for download, together with background information and lists of available products.
3. BACKGROUND INFORMATION ON WIND ACTION

General
3.1 The National Annex to BS EN 12899-1:2007 provides wind load values for the UK as an alternative to calculating actions using BS EN 1991. The wind pressure obtained using the table NA.2 (Appendix A) is conservative so, for large signs or where a number of signs are to be placed in a similar location, it is recommended that detailed calculations to BS EN 1991-1-4 be carried out.

3.2 The simplified method using the National Annex wind action values will take between ½ to ¾ hour depending on the complexity of the sign and location. This compares favourably with the alternative using BS EN 1991-1-4 to calculate the wind action, which can take several hours. The EN 1991-1-4 method is more complex and requires more detailed information about the location of the sign, which may not be readily available to the designer. It is possible to put much of the work into a spreadsheet to significantly reduce the design time, or to use suitable software that is available. It is also possible to calculate a wind load that can be safely applied to a whole scheme, authority or maintenance area to avoid repeating this work for every sign.

3.3 In the examples the partial action factor recommended in the 12899 National Annex (class PAF1) is applied and an additional factor $\gamma_f$ of 1.0 included. $\gamma_f$ is traditionally used to allow for the possibility of inaccurate assessment of the effects of actions and unforeseen stress distribution in the structure. For a simple cantilever sign structure the possibility of inaccurate assessment of the effects of actions is limited, and, where the posts have been verified by static load testing, unforeseen stress distribution in the structure and variations in dimensional accuracy achieved in construction are also unlikely. Similarly for steel sections the reliability of the product is considered equivalent to static load tests so a factor of 1.0 is proposed. A designer may use this factor to allow for the possibility of future modifications to the sign or the addition of small signs to the post.

3.4 Designers will be able to achieve more efficient designs using the more rigorous method but the proposed National Annex simplified method will minimise the design effort for all but the more complex situations.

3.5 The example calculations are set out to match the format of limit state design codes. Load action $\times$ load factors $\leq$ element capacity/material factor

3.6 There BS EN 1991-1-4 National Annex o methods should be used for calculating the exposure factor, $c_e(z)$, using a look-up table based on the distance from the shoreline. The wind action also depends on the height of the centroid of the sign from ground level. The wind loads presented in the 12899-1 National Annex Table NA.2 are based on this method, with the assumption of terrain category II. This terrain category is valid for all but the most exposed situations, but will be conservative for sheltered town sites.

Aerodynamic Force Coefficient, $c_f$
3.7 The National Annex to BS EN 12899-1 Table NA.2 (Appendix A) gives the force coefficient (also known as the shape factor) for elements with different aspect ratios. This is based on a review of the available guidance and how it should be applied to signs. EN 12899-1:2007 specifies the use of $c_f = 1.20$ (clause 5.3.1.1). This conflicts with advice in BS EN 1991-1-4 clause 7.4.3 which proposes a value of $c_f = 1.8$ for ‘signboards’. The value of 1.8 is very conservative for signs, and it is valid to use an alternative method that considers them as structural elements with sharp edges.

3.8 Two different methods were reviewed for calculating $c_f$ for elements with sharp edges. The first method uses BS EN 1991-1-4 Clause 7.7 and the other BS 6399 clause 2.7. The values in table NA2 in the
3.9 The BS EN 1991-1-4 Clause 7.7 method calculates $c_f = c_{f,o} \cdot \psi_\lambda$ with $c_{f,o} = 2$ for all cases. The end effect is calculated from clause 7.13, which uses table 7.16 and figure 7.36 to obtain the factor. Section 1 is valid for signs as all will be less than 15 m long. The rule in this case as $\lambda = 2 \cdot l/b$, which is where this method is more conservative than the National Annex. Fig 7.36 gives a reduction factor based on the aspect ratio with a solidity ratio of 1 being valid for signboards.

3.10 The National Annex to BS EN 1991-1-4 replaces table 7.16 of the main standard with table NA.6. This results in a change of formula for the slenderness ratio to: $\lambda = l/b$.

3.11 The limit values are as follows:

<table>
<thead>
<tr>
<th>Code</th>
<th>$l/b$</th>
<th>$\lambda$</th>
<th>$\psi_\lambda$</th>
<th>$c_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>EN 1991-1-4</td>
<td>1</td>
<td>2</td>
<td>0.63</td>
<td>1.26</td>
</tr>
<tr>
<td>NA to BS EN 1991-1-4</td>
<td>1</td>
<td>1</td>
<td>0.6</td>
<td>1.2</td>
</tr>
<tr>
<td>EN 1991-1-4</td>
<td>30</td>
<td>60</td>
<td>0.9</td>
<td>1.8</td>
</tr>
<tr>
<td>NA to BS EN 1991-1-4</td>
<td>60</td>
<td>60</td>
<td>0.9</td>
<td>1.8</td>
</tr>
</tbody>
</table>

Using this method no signs will have a $c_f$ close to 1.8.

3.12 It is important to note here that the reference height used in the design of elements is to the top of the element and not the centre of the area as it is for signs. This does not directly affect the calculations for $c_f$ but may be relevant when considering this method for use on signs.

**Design wind action to BS EN 1991-1-4**

3.13 The examples in Appendix C show the detailed method for calculating wind action to BS EN 1991-1-4 and its UK National Annex. The examples use the shortened formula NA.3a where orography is not significant and the factor for $c_{e(z)}$ is taken from table NA.7 for a known distance from the shoreline. Where orography is significant formula NA.4a/NA.4b should be used.

3.14 The wind speeds are defined by the wind map Figure NA.1 in the BS EN 1991-1-4 National Annex. The height to the centroid is defined in BS EN 1991-1-4 fig 7.21 and used in equation 4.8 in clause 4.5, which is then used in clause 5.3 equation 5.3. The wind force is therefore defined using the height to the centre of the sign. Note that equation 4.8 is replaced by equation NA.3 in clause NA 2.17 in the National Annex.

3.15 BS EN 1991-1-4 equation 5.3 clause 5.3 uses a structure factor $c_sc_d$ which is calculated in equation 6.1 in Clause 6.3.1. This factor is not used in the calculations as it is considered insignificant for traffic signs. The National Annex to BS EN 1991-1-4 clause NA 2.20 gives guidance on the calculation of $c_sc_d$, which for most signs will be near 1.0.

3.16 The examples do not include the orography factor calculated to clause NA 2.9 and Annex A3. This is rarely significant but should be included at the calculation of $q_p(z)$. For areas with potential for wind funnelling refer to section 6 for guidance.

**Wind Load values from the National Annex to BS EN 12899-1:2007**

3.17 The wind values given in BS EN 12899-1 Table NA.2 are based on calculations using both BS EN 1991-1-4 and its National Annex, and taking the more conservative result. The latter method has since been confirmed as the most appropriate for the UK. The height bands were reviewed to get the most efficient wind value for each region. The limit on overall sign height of 7 m was chosen because
HA standards require signs taller than this to have design and check certificate signed by a chartered engineer.

3.18 The wind values assume category II terrain roughness. Terrain categories are defined in BS EN 1991-1-4:2005, clause 4.3.2, and category II and higher are the predominate terrain roughness parameters in the country. The wind action values would be excessively conservative if they all were based on the higher category I or 0. The values given cover the majority of situations with limitations of: maximum altitude; wind velocity grouping, etc. If a particular site falls in terrain category I or 0 then detailed design will be required. Exposure factors are related to the height, the smaller the height the greater the effect of the terrain roughness (terrain category). The relationship is not one that can be simplified into a general rule.

3.19 The reference heights for the wind values in the National Annex are from ground level to the top of the sign, rather than to the centroid. If the height to the centroid of the sign(s) is greater than $\frac{3}{4} H$, then the limiting overall heights 4 m and 7 m in the table should be reduced to 3 m and 5.25 m respectively (unfortunately these heights are incorrectly stated as 3.25 m and 5 m in Note 2 to table NA.2 in the BS EN 12899-1:2007 National Annex).

3.20 The wind load values in table NA.2 are valid except in the following areas: the small area approximately 15 km to the south of Cape Wrath in north-west Scotland, Cornwall west of Falmouth, and Ynys Môn (Isle of Anglesey) in Wales. These areas should be treated as if they were entirely within 5 km of the shoreline.

3.21 The example calculations are based on the wind load values in the National Annex to BS EN 12899-1:2007. The values for $c_f$ in the lookup table in the National Annex are based upon the EN 1991-1-4 procedure for thin structural elements, which is an improvement on the conservative value of 1.8 recommended for ‘signboards’. The method used to calculate $c_f$ is in accordance with BS EN 1991-1-4 which is slightly more conservative than the method in its National Annex. In the example calculations we have not shown interpolation from the table for $c_f$ though it would be acceptable.

3.22 The table NA.2 gives a maximum sign height of 7 m, which is rarely exceeded in practice. For the higher signs the designer should use the more rigorous method, as on trunk roads these will require certification as Category 0 structures in accordance with BD 2/05.

3.23 The Wind Load values in Table NA.2 are subject to the following caveats:

- The values are only valid up to an altitude of 250 m.
- The values are only relevant where there is no local funnelling effect, or significant topographical features such as cliffs or escarpments (referred to as orography in clause NA 2.9 and Annex A.3 of BS EN 1991-1-4). Refer also to section 6.
4. BACKGROUND INFORMATION ON SUPPORT DESIGN

4.1 The Ultimate Limit State (ULS) effects are a combination of bending moment, shear and occasionally torsion. For the support design, these are generally all at maximum at the top of the connection to the foundation. At the Serviceability Limit State (SLS) the structure will not fail structurally but will be at the limit of performance requirements, such as acceptable deflection. For the SLS check a lower wind pressure is used and the partial action (safety) factor is 1.0.

4.2 The design effects are calculated from simple analysis of loads on a cantilever including the load factors.

4.3 The designer should have access to the characteristic properties of the proposed supports. These can be developed from first principles by calculating the section area, second moment of area, etc., or from the manufacturer’s published properties for individual products. The characteristic property is defined as that which 95% of the products will achieve (not the average). This is calculated by statistical analysis of test results when carried out. These values are reduced by a factor $\gamma_m$ (given in Table 7 of BS EN 12899:2007) to allow for the variability of the material. The selection of the support is based on the simple criterion that the capacity of the post must be greater than the load effect.

4.4 The example calculations take no account of wind pressure on the supports themselves, i.e. on the area below the sign plate that is exposed to wind. It has been shown that this has about 2% effect on the overall actions based on a height to the centroid of the sign, which is conservative. This is considered insignificant. If the designer wishes to include a value to allow for this it is suggested that use of $\gamma_f=1.05$ would be adequate.

4.5 Torsion may be significant where the sign plate is positioned eccentrically on a single post or where the sign is part shielded by an obstruction that eliminates wind action on part of it, or when a point load of 500 N (class PL3) on the extremity of the sign is considered. The use of an eccentricity of $0.25 \times$ the width of the sign (recommended in BS EN 1991-1-4 clause 7.4.3 (2)) is considered excessive. It is suggested that the maximum torsion is achieved with the maximum wind action on the area of sign plate one side of the post, and that the associated bending be calculated using wind on only this area and not the whole sign. Where torsion due to wind action is significant the support should be checked for the combined effect of torsion and bending.

4.6 The shear capacity check is given but in practice it is likely to be significant only for large signs at low mounting heights supported on passively safe posts.

4.7 The deflection calculation is different to the previous method to BS 873, as it is checked at the top of the sign where maximum deflection occurs, not at the centroid. As the acceptable deflection is expressed in millimetres per metre height, this makes little difference in practice. The limit on deflection may not be suitable for non-standard signs such as VMS and those with moving components. In this situation refer to the manufacturer for specific deflection limits.
5. FOUNDATION DESIGN

Design of spread foundations to EN 1990 and EN 1997

5.1 The design of spread foundations is based on the calculation of stability against sliding and overturning of the base and maximum bearing pressure, while ignoring all passive resistance of the soil. The design approaches detailed in EN 1997 uses factored actions from EN 1990. The GEO Design Approach 1 for combinations 1 and 2 must be applied in the UK. The resistance factors are in accordance with the National Annex to EN 1990. The factors for building structures are recommended because they are similar to historic sign designs and are less onerous than the alternative for bridges. Designs are considered safe if the resistance is greater than the actions. The Eurocode method does not use an overall factor of safety but the results equate to a slightly higher overall safety factor than designers will be familiar with. The following table shows the relevant factors for Eurocode method for building structures.

<table>
<thead>
<tr>
<th>Condition</th>
<th>EN 1990 – EQU Limit State</th>
<th>EN 1990 – GEO Design Approach 1 – Combination 1</th>
<th>EN 1990 – GEO Design Approach 1 – Combination 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\gamma_{G,\text{dstb}}$, $\gamma_{G,\text{stb}}$</td>
<td>$\gamma_F(\text{unfav})$, $\gamma_F(\text{fav})$</td>
<td>$\gamma_F(\text{unfav})$, $\gamma_F(\text{fav})$</td>
</tr>
<tr>
<td>Overturning</td>
<td>1.50 0.95</td>
<td>1.50 1.00</td>
<td>1.30 1.00</td>
</tr>
<tr>
<td>Sliding</td>
<td>1.50 -</td>
<td>1.30 -</td>
<td>-</td>
</tr>
</tbody>
</table>

*EN 1990 action factors for the design of sign bases*

<table>
<thead>
<tr>
<th>$W_k$ = Vertical actions</th>
<th>$V_k$ = Bearing pressure</th>
<th>$Y$ = Resultant soil reaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L$ = Length of foundation</td>
<td>$e$ = eccentricity of reaction</td>
<td>$M_{over}$ = Overturning actions</td>
</tr>
</tbody>
</table>

*Bearing pressure diagrams for spread foundation*
5.2 The strength of soils is usually based on the assumption of poor soils, as experience has shown that it is unusual for a ground investigation to be carried out prior to installing signs. A value of 100 kN/m$^2$ is recommended as a minimum for design on a typical highway embankment or cutting under wind loading conditions. It is good practice, but not essential, to aim for the resulting force to be within the middle third of the base (refer to diagram above). The equations for the alternative cases where the reaction is inside or outside the middle third will not be correct if the wrong one is chosen. For example, if $V_{\text{min}}$ is negative the reaction is outside the middle third. The most efficient base is achieved with minimum depth and width $W$ (parallel to the sign face), and increasing the length $L$ (perpendicular to the sign face) to achieve stability. Please note that the definitions of $W$ and $L$ are reversed from their usage in previous editions of this Guide.

5.3 It is recommended that spread foundations are reinforced to accommodate bending effects, and pull-out of anchorages where used. Reinforcement may be omitted for smaller signs and squarer bases where it can be shown that tension due to bending within the concrete is not significant.

**Design of planted foundations to BD 94/07**

5.4 Planted foundations are based upon the method in the Highways Agency standard BD 94/07, which was developed from earlier empirical methods and is similar to the procedure used for lighting column bases. The design uses unfactored actions and resistance and applies a defined factor of safety of 1.25 for overturning of the foundation to the destabilising action. The stability of this type of foundation relies on the passive resistance of the soil surrounding the foundation which must be well compacted. For this reason the use of planted foundations on a slope or near ditches or excavations require special consideration as discussed below. Regardless of the orientation of the sign face, the effect of a slope should always be considered, but it becomes increasingly important the closer the sign plate is to perpendicular to the line of greatest slope.

5.5 This design method is only appropriate for foundations where the depth is significant in relation to the diameter (say twice the diameter) in order to mobilise the passive resistance of the soil. Minimum planting depth requirements are given in BS EN 40-2 Table 7. BD 94/07 clause 11.3 recommends using the middle column of planting depth values from Table 7 for traffic signs and signals, but allows a reduced depth of 600 mm for signs under 2.0 m total height. Precautions should be taken when constructing planted foundations to ensure that they do not damage any statutory undertaker’s service.

**Planted foundations on a slope**

5.6 Planted foundations on a slope can be designed using a simple conservative approach based on the assumption that the top layers of soil are not effective. Refer to Figure 5.2. Two alternative methods are proposed to define a Notional Ground Level for calculating actions and resistance. The calculations are otherwise identical to those for a normal planted foundation, but using a greater height of post and a reduced planting depth. The first method is based on providing a minimum horizontal distance of 3 m to the nearest edge of the slope and the second method on ignoring a proportion of the total foundation depth, the percentage varying with the slope angle. The slope angle should be the highest value on the downhill side of the foundation within 3 m of the post. The slope of the ground above the position of the sign is not significant and can be ignored.

5.7 When planting a foundation on a slope there is an additional risk of causing temporary or long-term instability to the slope. In cohesive material the excavations may introduce a water path into the slope, which could lead to failure.

5.8 Planted foundations for signs on slopes will have significantly deeper planting depths and greater access problems. A planted foundation in these cases may not be achievable and will pose a greater risk to site operations, so it may be more appropriate to provide a standard spread foundation. Another alternative would be to auger a concrete base and provide a socket or bolted connection for the post. Careful consideration of the need for reinforcement is required in this case.
**Figure 5.1** $F_{\text{slope}}$ for determining $h_b$ using Method 2

- Angle of slope $\alpha$
- Mounting height above ground $h_m$
- Width of sign face $l$
- Height of sign face $b$
- Total height $= h_m + b = H$
- Height to centroid of sign area $= z$
- $h_m + b/2$
- Effective depth of post buried above foundation $h_b$

**Figure 5.2**

Planted foundations on a slope
Calculation of revised $h_b$ for sign on slope:

Method 1  \hspace{1cm} h_b = 3 \text{ metres} \times \tan \alpha

Method 2  \hspace{1cm} h_b = F_{\text{slope}} \times P \quad \text{where } F_{\text{slope}} \text{ is read from Figure 5.1}

For either method:

Foundation support planted in plain concrete:

- minimum diameter of foundation in the ground: $D$
- planting depth of foundation: $P$ \hspace{0.5cm} (\geq 2D)
- Effective planting depth $P_{\text{eff}} = P - h_b$

$P_{\text{eff}}$ to be used in calculations in place of $P$. See Appendix C Example 1 section 1.5

5.9 The parameters can be used for the design of planted foundations as illustrated in Appendix C example 1 section 1.5. Note that $P_{\text{eff}}$ should be used in place of $P$ in all parts of the calculation, including the minimum planting depth from EN 40 and depth to diameter ratio.

5.10 If example 1 in Appendix C (section 1.5) is reworked with a 15° slope, the total planting depth increases from 1.0 to 1.81 (using method 1), or to 1.54 (using method 2). Method 2 is less conservative in this case but involves more iteration.
6. WIND FUNNELLING AND TOPOGRAPHY

6.1 At present there is no published guidance available for design in areas of wind funnelling. A study is in progress to define where funnelling occurs and to determine simple procedures to use in these situations. This section provides a summary of the current findings and recommendations.

6.2 Wind direction may be significantly changed by topography so it is recommended that the wind direction factor be ignored for the design of minor structures. The direction factor $c_{dir}$ of BS EN 1991-1-4 should be set to 1.0 for the design of all minor structures.

6.3 The effect of all two-dimensional features such as embankments and escarpments should be included irrespective of the orientation of a sign mounted on it. The effect of an embankment can either be allowed for by using an orography factor or by adding the height of the embankment to the sign height for calculation of wind speed.

6.4 Funnelling may be neglected for traffic signs of height 5 m or less where they are designed to the simplified wind pressures of the National Annex to BS EN 12899-1.

6.5 The standard for minor structures, BD 94/07 should be clarified as follows:
- Clause 4.1: the reference to BS EN 40 for design of lighting columns should be supplemented by reference to PD 6547.
- Clause 4.4: the definition of “sites subject to significant local wind funnelling” includes sites where steep-sided valleys or cuttings are present, or where a highway runs along a steep-sided slope (i.e. funnelling around the side of a hill). It is not necessary for a valley to be narrowing for funnelling to occur, nor for the valley to be parallel to the roadway – funnelling may be caused by gullies running transversely to the road alignment.

6.6 Wind funnelling should be considered where three-dimensional topography occurs and a simple orography approach cannot be used. In the absence of detailed guidance, the size of significant features may be assessed using methods for orography in EN 1991 clause 4.3.4, A3 and Fig NA.2

6.7 In the absence of comprehensive guidance on funnelling the following interim advice is given, subject to possible change as a result of ongoing research. Where a structure is in a site subject to possible wind funnelling, the topography factor $c_o(z)$ of BS EN 1991-1-4 (factor $f$ of BS EN 40-3-3) shall be a minimum of 1.4 at heights up to 5 m, reducing to 1.2 at heights of 10 m and above (with linear interpolation between). Where funnelling features exist in combination with other significant topography (e.g. gullies cut into a steep hillside), consideration should be given to adopting a higher topography factor of 1.65 at heights up to 5 m, reducing to 1.2 at heights of 10 m and above.

6.8 If the location is more complex, expert advice must be sought.
## APPENDIX A: Table NA.2 of National Annex

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### Recommended classes or values for physical performance most suitable for UK practice

<table>
<thead>
<tr>
<th>Property</th>
<th>Recommended performance class or value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>EITHER</strong> design to BS EN 1991-1-4 using the 10 minute mean wind reference speed appropriate to the locality of the sign, taken from the national wind map (shown in BS EN 1991-1-4 UK National Annex) and adjusted for altitude.</td>
<td></td>
</tr>
<tr>
<td><strong>OR</strong> use the appropriate wind load value taken from the table below, which has been calculated using BS EN 1991-1-4.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>Sign maximum overall height H (m)</th>
<th>Distance from the shoreline (d)</th>
<th>Wind load value (kN m$^{-2}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>See Note 2</td>
<td>d ≤ 5 km</td>
<td>d &gt; 5 km</td>
</tr>
<tr>
<td>England</td>
<td>4.0</td>
<td>1.1</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>7.0</td>
<td>1.3</td>
<td>1.2 *</td>
</tr>
<tr>
<td>Wales</td>
<td>4.0</td>
<td>1.1</td>
<td>1.0 *</td>
</tr>
<tr>
<td></td>
<td>7.0</td>
<td>1.4</td>
<td>1.3</td>
</tr>
<tr>
<td>Northern Ireland &amp; Isle of Man</td>
<td>4.0</td>
<td>1.3</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>7.0</td>
<td>1.6</td>
<td>1.4 *</td>
</tr>
<tr>
<td>Scottish mainland</td>
<td>4.0</td>
<td>1.5</td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td>7.0</td>
<td>1.8</td>
<td>1.6 *</td>
</tr>
<tr>
<td>Scottish islands</td>
<td>4.0</td>
<td>1.6</td>
<td>1.6</td>
</tr>
<tr>
<td></td>
<td>7.0</td>
<td>2.0</td>
<td>1.9</td>
</tr>
</tbody>
</table>
Table NA.2 (continued)

<table>
<thead>
<tr>
<th>Property</th>
<th>Recommended performance class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Partial safety factors</td>
<td>Class PAF 1, Table 6.</td>
</tr>
<tr>
<td>Partial material factors</td>
<td>According to the material used in the manufacture of the sign, see Table 7.</td>
</tr>
</tbody>
</table>

**NOTE 1** The wind load values marked * above are not valid in the following areas: the small area approximately 10 miles to the south of Cape Wrath in North West Scotland, Cornwall west of Falmouth, and Ynys Môn (Isle of Anglesey) in Wales. For these areas the wind load value for d ≤ 5 km from the shoreline should be used.

**NOTE 2** The sign maximum overall height H for the wind load value is measured from ground level to the top of the sign assembly and not to the centroid of the sign. If the height to the centroid of the sign or signs is greater than ¾ H then the maximum overall height of 4 m and 7 m in the table above should be changed to 3.0 m and 5.25 m respectively, or structural design should be to BS EN 1991-1-4:2005.

**NOTE 3** All the wind load values given in the above table apply up to a limiting altitude of 250 m above sea level (at ground level). Above this altitude, structural design of all signs should be to BS EN 1991-1-4:2005.

**NOTE 4** The wind load values in the table above are based on a wind speed return period of 25 years. Highway Authorities may specify a return period of 50 years, when designing signs to BS EN 1991-1-4:2005.

### Table showing force coefficients \( c_f \) to be used in accordance with section 7.7 of BS EN 1991-1-4:2005.

<table>
<thead>
<tr>
<th>Aspect ratio</th>
<th>1</th>
<th>1.6</th>
<th>3</th>
<th>5.5</th>
<th>7.5</th>
<th>13.5</th>
<th>20</th>
<th>30</th>
</tr>
</thead>
<tbody>
<tr>
<td>Force coefficient ( c_f )</td>
<td>1.26</td>
<td>1.3</td>
<td>1.35</td>
<td>1.4</td>
<td>1.5</td>
<td>1.6</td>
<td>1.7</td>
<td>1.8</td>
</tr>
</tbody>
</table>

**NOTE 5** The Aspect ratio is the larger of \( l/b \) or \( b/l \) where \( b \) is the height and \( l \) is the width of the sign face (including any backing board and light spill screen).

**NOTE 6** The overall safety factor should be obtained by multiplying the partial action factor and partial material factor given in Table NA.2.

**NOTE 7** It is recommended that for very exposed sites, or sites subject to local funnelling effects, signs should be designed to BS EN 1991-1-4:2005.

**NOTE 8** The wind load values given above are conservative. Highway Authorities may wish to derive a wind load value, for a specific location or defined area, by using BS EN 1991-1-4: 2005.

**NOTE 9.** Where the material properties or method of jointing are not known, the designer should select the highest value for partial material factor.

**NOTE 10** The wind load on the sign structure is obtained by multiplying the wind pressure (the wind load value in the table above) by the force coefficient \( c_f \) and the overall safety factor.

**NOTE 11** The eccentricity should normally be zero and the force coefficient \( c_f \) should be taken from the table above, unless calculated in accordance with section 7.7 of BS EN 1991-1-4:2005.
### Table NA.2 (continued)

<table>
<thead>
<tr>
<th>Property</th>
<th>Recommended performance class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Point loads</td>
<td>Class PL3, Table 10</td>
</tr>
<tr>
<td>Dynamic snow loads</td>
<td></td>
</tr>
<tr>
<td>If snow blowers or snow ploughs are not regularly used</td>
<td>Class DSL0, Table 9</td>
</tr>
<tr>
<td>If snow blowers are regularly used</td>
<td>Class DSL1, Table 9</td>
</tr>
<tr>
<td>If snow ploughs are regularly used</td>
<td></td>
</tr>
<tr>
<td>Ploughing speed of 40 mph or less</td>
<td>Class DSL2, Table 9</td>
</tr>
<tr>
<td>Ploughing speed greater than 40 mph</td>
<td>Class DSL4, Table 9</td>
</tr>
</tbody>
</table>

**NOTE 12** It is for the highway authority to decide whether to include snow loading in design by selecting a class other than DSL0. This will usually only be necessary in locations where there is considered to be a significant problem of damage to signs during snow clearing operations.

**NOTE 13** If snow blowers are correctly aligned while in use the load on the sign should be minimal.

<table>
<thead>
<tr>
<th>Temporary deflection of sign plates and supports</th>
<th>Use the appropriate temporary deflection bending class, and temporary deflection torsion class given in the table below.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Product</td>
<td>Recommended performance class</td>
</tr>
<tr>
<td>---------</td>
<td>-------------------------------</td>
</tr>
<tr>
<td>Bending class</td>
<td>Torsion class</td>
</tr>
<tr>
<td>Sign plate</td>
<td>TDB4, Table 11</td>
</tr>
<tr>
<td>Support – not passively safe (Class 0 in BS EN 12767)</td>
<td>TDB4, Table 11</td>
</tr>
<tr>
<td>Support – passively safe (compliant with a performance class from BS EN 12767)</td>
<td>TDB5, Table 11</td>
</tr>
</tbody>
</table>

**NOTE 14** The deflection of the sign plates should be evaluated relative to the supports.

<table>
<thead>
<tr>
<th>Property</th>
<th>Recommended performance class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Piercing of sign face</td>
<td>Class P3, Table 13</td>
</tr>
<tr>
<td>Edging of sign plates</td>
<td>Class E1, Table 14</td>
</tr>
<tr>
<td>Corrosion protection</td>
<td>Class SP1 or SP2, Table 15</td>
</tr>
</tbody>
</table>
APPENDIX B: Flowchart for Determining Wind Load

1: Calculate aspect ratio and look up $c_f$ (Table NA 2)
2: Look up partial action factors (Table NA 2)
3: Calculate Wind Actions
4: Look up partial material factors (Table NA 2)
5: Calculate post resistance, using factored strength & properties.

Start

Is the site exposed / subject to local funneling?

Y


N

Height to Centroid $\leq \frac{3}{4} H$?

Y

Is Height $\leq 4 m$?

Y

Look up wind class using Table NA 2 $H=4 m$ for location and distance from shoreline

N

Is Height $\leq 7 m$?

Y

Look up wind class using Table NA 2 $H=7 m$ for location and distance from shoreline

N

Are wind load actions $< post$ resistance?

Y

Finish. Proceed to foundation design.

N

Are wind load actions $\leq post$ resistance?

Y

Proceed to foundation design.

N
APPENDIX C: EXAMPLES

Introduction
Two examples are examined: a small circular sign on a single support with a planted foundation and a rectangular sign on two supports with a more conventional spread foundation.

For each sign, the wind load is determined using both the simplified method of the National Annex to BS EN 12899-1 (sections 1.1 and 2.1), and the more rigorous alternative method of BS EN 1991-1-4 (sections 1.2 and 2.2). This enables the two approaches to be compared for complexity and results. Whichever method is used to obtain it, the basic wind action is converted into a wind force for each of the states to be examined (sections 1.3 and 2.3). These are used in sections 1.4 and 2.4 for checking the sign supports. Finally, in sections 1.5 and 2.5, the chosen foundation is assessed.

EXAMPLE 1: a circular sign with a planted foundation

Location: Surrey, England, 10 km from the coast, not on a very exposed site, cliff or escarpment, nor at a site subject to wind funnelling.

Mounting height above ground \( h_m \) 2.0 m
Width of sign face \( l \) 0.9 m
Height of sign face \( b \) 0.9 m
Total height \( H \) 2.9 m
Height to centroid of sign area \( z \) 2.45 m
Depth of post buried above foundation \( h_b \) 0
Foundation: support planted in plain concrete:
minimum diameter of foundation in the ground \( D \)
planting depth of foundation \( P \) \( \geq 2D \)
Example 1 Section 1: Basic Wind Action using BS EN 12899 National Annex

1.1.1 Look up wind load for site

\[ H = 2.9 \text{ m height, therefore Basic Wind Pressure, } w_b = 1.0 \text{ kN/m}^2 \]

(Used in section 1.3 below.)

Table NA 2

Example 1 Section 2: Basic Wind Action using BS EN 1991-1-4

1.2.1 Look up fundamental value of basic wind velocity

References relate to BS EN 1991-1-4 and its National Annex

\[ v_{b,0} = v_{b,\text{map}} \cdot c_{\text{alt}} = 21.5 \times 1.25 = 26.88 \text{ m/s} \]

Eqn NA1

where

\[ v_{b,\text{map}} = \text{value of the fundamental basic wind velocity before the altitude correction is applied. (21.5 m/s selected from map)} \]

\[ c_{\text{alt}} = \text{altitude factor} \]

\[ c_{\text{alt}} = 1 + 0.001 \cdot A = 1 + 0.001 \times 250 = 1.25 \]

\[ A = \text{Altitude of the site (m) above mean sea level (250 m in this example)} \]

Figure NA1

Eqn NA2a

1.2.2 Assess Terrain Orography

Not very exposed site on cliff/escarpment or in a site subject to local wind funnelling. Therefore \( c_o = 1.0 \)

Fig NA2

1.2.3 Determine Design Life Requirement

\[ c_{\text{prob}} = \left( \frac{1 - K \cdot \ln(-\ln(1 - p)))}{1 - K \cdot \ln(-\ln(0.98))} \right)^n = 0.96 \]

Eqn 4.2

where

\[ p = \text{design annual probability of exceedence} \]

\[ p = 1/\text{design life} = 1/25 = 0.04 \text{ (for signs design life is 25 years)} \]

\[ K = \text{Shape parameter} = 0.2 \]

\[ n = \text{exponent} = 0.5 \]

NA 2.8

1.2.4 Basic Wind Velocity

\[ v_b = c_{\text{dir}} \cdot c_{\text{season}} \cdot v_{b,0} \]

Eqn 4.1

\[ v_b = 1.0 \times 1.0 \times 26.88 = 26.88 \text{ m/s} \]

where

\[ c_{\text{dir}} = \text{directional factor} = 1.0 \]

\[ c_{\text{season}} = \text{season factor} = 1.0 \]

NA 2.6

NA 2.7
10 minute mean wind velocity having probability $p$ for an annual exceedence is determined by:

\[ v_{b,25\text{ years}} = v_b \cdot c_{\text{prob}} \]

\[ v_{b,25\text{ years}} = 26.88 \times 0.96 = 25.80 \text{ m/s} \]

1.2.5 **Basic Velocity Pressure**

\[ q_b = \frac{1}{2} \rho \cdot v_b^2 = 0.5 \times 1.226 \times 25.80^2 = 0.408 \text{ kN/m}^2 \]

where \( \rho \) = air density = \( 1.226 \text{ kg/m}^3 \)

1.2.6 **Peak Velocity Pressure**

\[ c_{\text{e,flat}}(2.45) = 1.66 \]

\[ q_p(z) = c_{\text{e,flat}}(z) \cdot q_b = 1.66 \times 0.408 = 0.68 \text{ kN/m}^2 \]

For \( z = 2.45 \text{ m} \) where orography is not significant (\( c_o = 1.0 \)) and country terrain category

1.2.7 **Basic Wind Pressure**

\[ w_b = q_p(z) = 0.68 \text{ kN/m}^2 \]

This falls within BS EN 12899-1:2007 class WL3, but use of these classes is not recommended.

---

**Example 1 Section 3: Wind Force (for either method)**

1.3.1 **Determine Force Coefficient** $c_f$

\[ \lambda = \text{effective slenderness ratio of sign or aspect ratio} \]

\[ \lambda = \frac{l}{b} \text{ or } \frac{b}{l} = 0.9/0.9 = 1.0 \]

Therefore $c_f = 1.26$

1.3.2 **Calculate Total Wind Force**

\[ F_w = c_f \cdot c_d \cdot c_t \cdot q_p(z_e) \cdot A_{\text{ref}} \] \( (A_{\text{ref}} = \text{area of sign}) \)

$c_c \cdot c_d$ for signs can be taken as 1.0

\[ q_p(z_e) = w_b \] \( (\text{the basic wind pressure determined above}) \)

\[ F_w = c_t \cdot w_b \cdot A_{\text{ref}} \]

Use $w_b = 1.0 \text{ kN/m}^2$ from section 1.1.1 above for this example.

\[ F_w = 1.26 \times 1.0 \pi \left( \frac{0.9}{2} \right)^2 = 0.80 \text{ kN} \]
1.3.3 Identify Partial Action Factors $\gamma_F$

- ULS (bending and shear) $\gamma_F = 1.35$  
- SLS (deflection) $\gamma_F = 1.0$  

Additional factor $\gamma_{f3}$ taken as 1.0, but could alternatively be 1.1  

1.3.4 Calculate Design Wind Force on the sign

\[
F_{w,d} = F_w \cdot \gamma_F \cdot \gamma_{f3}
\]

\[
F_{w,d} \text{(ULS)} = 0.80 \times 1.35 \times 1.0 = 1.08 \text{ kN}
\]

\[
F_{w,d} \text{(SLS)} = 0.80 \times 1.0 \times 1.0 = 0.80 \text{ kN}
\]

1.3.5 Wind Force Equivalent to 1-year Return Period

The wind velocity for calculating the temporary deflection (SLS) criterion is 75% of the reference wind velocity, as it is based upon a 1 year mean return period. The 0.96 factor below reverses the $c_{\text{prob}}$ conversion from 50 to 25 year return period used above (in 1.2.3).

\[
F_{w,d} \text{(1 year)} = F_{w,d} \text{(SLS)} \times \frac{0.75^2}{0.96^2} = 0.80 \times \frac{0.75^2}{0.96^2} = 0.488 \text{ kN}
\]

If the EN 1991-1-4 Basic Wind Pressure from section 1.2.7 above is used, the calculated values are:

- $F_{w,d} \text{(ULS)} = 0.74 \text{ kN}$
- $F_{w,d} \text{(SLS)} = 0.55 \text{ kN}$
- $F_{w,d} \text{(1 year)} = 0.34 \text{ kN}$

---

Example 1 Section 4: Support design (for either method)

1.4.1 Ultimate Design Action

\[F_{w,d} \text{(ULS)} = 1.08 \text{ kN}\]

1.4.2 Ultimate Action Effects (wind action)

Ultimate design bending moment per post, $M_d$

\[
M_d = \text{Wind force} \times \text{lever arm to foundation} / \text{number of posts}
\]

\[
= F_{w,d} \text{ (ULS)} \cdot (z + h_b) / n \quad \text{(where } n = \text{number of posts)}
\]

\[
= 1.08 \times (2.45 + 0) / 1 = 2.65 \text{ kNm}
\]

Ultimate design shear per post, $V_d = \text{Wind force} / \text{number of posts}$

\[
V_d = F_{w,d} \text{ (ULS)} / n = 1.08 / 1 = 1.08 \text{ kN}
\]
1.4.3 Ultimate Action Effects (point load)

Investigate the effects of a 0.5 kN point load applied to the post:

**BS EN 12899-1**

**Tables NA 2 & 10**

**(class PL3)**

Applied Moment:

\[ M_d = 0.5 \times (H + h_b) \]

\[ M_d = 0.5 \times (2.9 + 0) \]

\[ M_d = 1.45 \text{kNm} \quad \text{(note this is less critical than } M_d \text{ from wind action)} \]

Applied Shear:

\[ V_d = 0.5 \text{kN} \]

Torsion = 0.5 \( \frac{L}{2} \)

\[ \text{Torsion} = 0.5 \times 0.900 = 0.225 \text{kNm} \]

1.4.4 Support Properties

Circular Hollow Section, CHS 88.9 \( \times \) 4.0 (S355 steel) is to be tried.

**Steelwork design guide**

Characteristic member capacities

\[ M_c = 9.24 \text{kNm} \]

\[ P_v = 137.0 \text{kN} \]

1.4.5 Partial Material Factor for Sign Support

\[ \gamma_m = 1.05 \text{ for steel sections (elongation > 15%)} \]

**BS EN 12899-1**

**Table 7**

1.4.6 Ultimate Capacity Check

Ultimate bending capacity = \( \frac{M_c}{\gamma_m} = 9.24 / 1.05 = 8.80 \text{kNm} \)

8.80 kNm > 2.65 kNm \( \text{OK} \)

Ultimate shear capacity = \( \frac{P_v}{\gamma_m} = 137.0 / 1.05 = 130.48 \text{kN} \)

130.48 kN > 1.08 kN \( \text{OK} \)

1.4.7 Combined Bending and Torsion

\[ \frac{M}{M_u} + \frac{T}{T_u} \leq 1 \]

By inspection, the post sections have sufficient spare capacity to resist combined bending and torsion, since only 50% of available capacity of the post is utilised.
1.4.8 Temporary Deflection Calculation

\[ F_{w,d \ (1\ year)} = 0.488 \text{kN} \]

Uniformly distributed load along sign face = \( F_{w,d \ (1\ year)} / b \)

\[ F_{w,d \ (1\ year)} / b = 0.488 / 0.9 \quad = 0.54 \text{kN/m} \]

where \( b \) = height of the sign face

Maximum deflection at top of sign (bending), \( \delta \)

\[ \delta = \frac{F_{w,d \ (1\ year)} / b}{24EI \cdot n} \left[ 3(H + h_b)^4 - 4(h_m + h_b)^3(H + h_b) + (h_m + h_b)^4 \right] \]

where

- \( E \) = Elastic modulus of structural steel, taken as \( 205 \times 10^3 \text{N/mm}^2 \)
- \( I \) = Moment of inertia
  \( = 96.3 \text{cm}^4 \) (for CHS 88.9 \( \times \) 4.0) \( = 96.3 \times 10^4 \text{mm}^4 \)
- \( n \) = number of posts
- \( H, h_b, \) and \( h_m \) as illustrated on diagram

\[ \Rightarrow \delta = \frac{0.54}{24 \times 205 \times 10^3 \times 96.3 \times 10^4 \times 1} \times \left[ 3 \times (2900 + 0)^4 - 4 \times (2000 + 0)^3 \times (2900 + 0) + (2000 + 0)^4 \right] \]

\[ \delta = 15.43 \text{mm} \]

Deflection per linear metre, \( \delta' = \frac{\delta}{H + h_b} = \frac{15.43}{(2.9 + 0)} = 5.32 \text{mm/m} \]

Maximum temporary deflection taken as TDB4 = 25 mm/m

25 mm/m > 5.32 mm/m \( OK \)

(If EN 1991-1-4 method is used, the calculated deflection is 3.74mm/m)

1.4.9 Conclusion

CHS 88.9 \( \times \) 4.0 (S355) is sufficient for the design.

A smaller section might also be suitable.

Notes:

Design forces may also include the moment and shear force from the wind action on the post(s).
The SLS deflection limits may be more onerous for signs with moving parts.
Example 1 Section 5: Planted Foundation Design to BD 94/07

1.5.1 Un-factored Design Action

\[ F_{w,d} \text{ (SLS)} = 0.80 \text{ kN} \]  
*Section 1.3.4 above*

1.5.2 Ground Resistance Moment, \( M_g \)

Minimum planting depth, \( P_{\text{min}} = 0.8 \text{ m} \)

\[ M_g = \frac{G \cdot D \cdot P^3}{10} \]

where

- \( G \) is a factor dependent on the ground in which the support is planted (in kN/m\(^2\) per m). Refer to BD 94/07 Table 3 for typical values of \( G \).
- \( D \) is the minimum diameter of the support and its surrounding concrete in the ground (in m).
- \( P \) is the planting depth (in m).

Try foundation design: \( D = 0.4 \text{ m}, P = 1.0 \text{ m} \)

\[ P \geq 2D \quad \text{OK} \quad \text{(to ensure the foundation will behave as a planted one.)} \]

Assume ‘Poor’ ground conditions, \( \therefore G \) taken as 230 kN/m\(^2\) per m

\[ \Rightarrow M_g = \frac{230 \times 0.4 \times 1.0^3}{10} = 9.20 \text{ kNm} \]

1.5.3 Destabilising Moment

The destabilising moment is calculated about a fulcrum point located at \( 1/\sqrt{2} \) of the planting depth below ground.

\[ \therefore \text{ lever arm} = [z + h_b + (\frac{1}{\sqrt{2}} P)] \]

\( z, h_b \) and \( P \) are as defined previously

\[ M_{DS} = F_{w,d} \times \text{lever arm to fulcrum point} / \text{number of posts} \]

\[ = F_{w,d} \times \frac{[z + h_b + (\frac{1}{\sqrt{2}} P)]}{n} \]

\[ = 0.80 \times (2.45 + 0 + \frac{1}{\sqrt{2}} \times 1.0)/1 = 2.53 \text{ kNm} \]

\[ \gamma_{s,d} \times M_{DS} = 1.25 \times 2.53 = 3.16 \text{ kNm} \]

where \( \gamma_{s,d} \) is the model factor, 1.25

*BD 94/07 clause 11.4*
1.5.4 Capacity Check

\[ M_g = 9.20 \text{ kNm} > \gamma_{sd} \times M_{DS} = 3.16 \text{ kNm} \quad OK \]

BD 94/07  
clause 11.6

1.5.5 Conclusion

A foundation 0.4 m diameter with 1.0 m planting depth is satisfactory.

A smaller diameter of foundation may also be suitable.

Note:

This design method does not apply to foundations on slopes, where the stability of the ground needs to be taken into account. In such instances, specialist geotechnical advice should be sought.

BD 94/07  
Clause 11.1
EXAMPLE 2: a rectangular sign with a spread foundation

Location Londonderry, Northern Ireland 7 km from the coast, not on a very exposed site on cliff / escarpment, nor in a site subject to wind funnelling

- Mounting height above ground: \( h_m = 1.5 \) m
- Breadth of sign face: \( l = 4.0 \) m
- Height of sign face: \( b = 2.5 \) m
- Total height: \( H = h_m + b = 4.0 \) m
- Height to centroid of sign area: \( z = h_m + b/2 = 2.75 \) m
- Depth of post buried above foundation: \( h_b = 75 \) mm
- Spread foundation:
  - Width of foundation parallel to sign: \( W \)
  - Length of foundation perpendicular to sign: \( L \)
  - Thickness of foundation: \( T \)

Example 2 Section 1: Basic Wind Action using BS EN 12899 National Annex

2.1.1 Look up wind action for site

\[ H = 4.0 \text{ m}, \text{ therefore Basic Wind Pressure}, w_b = 1.2 \text{ kN/m}^2 \]

EN 12899 Table NA.2
Example 2 Section 2: Basic Wind Action using BS EN 1991-1-4

2.2.1 Look up fundamental value of basic wind velocity

\[ v_{b,0} = v_{b,\text{map}} \cdot c_{\text{alt}} = 26.25 \times 1.217 = 31.95 \text{ m/s} \]

where
\[ v_{b,\text{map}} = \text{value of the fundamental basic wind velocity before the altitude correction is applied. (26.25 m/s selected from map)} \]
\[ c_{\text{alt}} = 1 + 0.001A = 1 + 0.001 \times 217 = 1.217 \]
\[ A = \text{Altitude of the site above mean sea level (217 m in this example)} \]

2.2.2 Assess Terrain Orography

Not very exposed site on cliff/escarpment or in a site subject to local wind funnelling. Therefore \( c_o = 1.0 \)

2.2.3 Determine Design Life Requirement

\[ c_{\text{prob}} = \left( \frac{1 - K \cdot \ln\left(\frac{\ln(1 - p)}{\ln(0.98)}\right)}{1 - K \cdot \ln(\ln(0.98))} \right)^n = 0.96 \]

where
\[ p = \text{design annual probability of exceedence} \]
\[ p = 1/\text{design life} = 1/25 = 0.04 \text{ (for signs design life is 25 years)} \]
\[ K = \text{Shape parameter} = 0.2 \]
\[ n = \text{exponent} = 0.5 \]

2.2.4 Basic Wind Velocity

\[ v_b = c_{\text{dir}} \cdot c_{\text{season}} \cdot v_{b,0} \]
\[ v_b = 1.0 \times 1.0 \times 31.95 = 31.95 \text{ m/s} \]

where
\[ c_{\text{dir}} = \text{directional factor} = 1.0 \]
\[ c_{\text{season}} = \text{season factor} = 1.0 \]

10 minute mean wind velocity having probability P for an annual exceedence is determined by:

\[ v_{b,25 \text{years}} = v_b \cdot c_{\text{prob}} \]
\[ v_{b,25 \text{years}} = 31.95 \times 0.96 = 30.67 \text{ m/s} \]
2.2.5 **Basic Velocity Pressure**

\[ q_b = \frac{1}{2} \rho \cdot v_b^2 = 0.5 \times 1.226 \times 30.67^2 = 0.577 \text{ kN/m}^2 \]  

*Eqn 4.10*

where \( \rho \) = air density = 1.226 kg/m³

**NA 2.18**

2.2.6 **Peak Velocity Pressure**

\[ c_e(2.75) = 1.74 \]  

*Fig NA7*

\[ q_p(z) = c_e(z) \cdot q_b = 1.74 \times 0.577 = 1.00 \text{ kN/m}^2 \]  

*Eqn NA 3a*

For \( z = 2.75 \text{m} \) where orography is not significant (\( c_o = 1.0 \)), country terrain category, flat and \( \geq 5 \text{km} \) from the shore.

**NA 2.17**

2.2.7 **Basic Wind Pressure**

\[ w_b = q_p(z) = 1.00 \text{ kN/m}^2 \]

(This is equivalent to BS EN 12899-1:2007 class WL5, but use of these classes is no longer recommended.)

---

### Example 2 Section 3: Wind Force (for either method)

2.3.1 **Determine force coefficient**

\[ \lambda = \text{effective slenderness ratio of sign or aspect ratio} \]

\[ \lambda = \frac{l}{b} = 4.0 / 2.5 = 1.6 \]

Therefore \( c_f = 1.30 \)

**BS EN 12899**

**Table NA 2**

2.3.2 **Calculate Total Wind Force**

\[ F_w = c_c c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref} \]  

*\( A_{ref} = \text{area of sign} \)*

*5.3 of EN 1991-1-4*

*3.15 of this Guide*

\[ c_c c_d \text{ for signs can be taken as 1.0} \]

\[ q_p(z_e) = w_b \]

\( w_b \) is the basic wind pressure from section 2.1.1 or 2.2.7 above. The section 2.1.1 value of 1.2 kN/m² is used for this example.

\[ F_w = c_f \cdot w_b \cdot A_{ref} \]

\[ F_w = 1.30 \times 1.2 \times 4.0 \times 2.5 = 15.6 \text{ kN} \]

2.3.3 **Identify Partial Action Factors** \( \gamma_F \)

- ULS (bending and shear) \( \gamma_F = 1.35 \)
- SLS (deflection) \( \gamma_F = 1.0 \)

**EN 12899**

**Table NA 2 & Table 6**

*(class PAF1)*
Additional factor $\gamma_f$, taken as 1.0, but could alternatively be 1.1.
For stability for spread foundation design refer to commentary above.

### 2.3.4 Calculate Design Wind Force on the sign

\[ F_{w,d} = F_w \cdot \gamma_F \cdot \gamma_f \]
\[ F_{w,d} \text{ (ULS)} = 15.6 \times 1.35 \times 1.0 = 21.1 \text{ kN} \]
\[ F_{w,d} \text{ (SLS)} = 15.6 \times 1.0 \times 1.0 = 15.6 \text{ kN} \]

### 2.3.5 Wind Force Equivalent to 1-year Return Period

The wind velocity for calculating the temporary deflection (SLS) criterion is 75% of the reference wind velocity, as it is based upon a 1 year mean return period. The 0.96 factor below reverses the $c_{\text{prob}}$ conversion from 50 to 25 year return period used above (in 2.2.3).

\[ F_{w,d} \text{ (1 year)} = F_{w,d} \text{ (SLS)} \times \frac{0.75^2}{0.96^2} = 15.6 \times \frac{0.75^2}{0.96^2} = 9.52 \text{ kN} \]

If EN 1991-1-4 method is used, the calculated values are:
\[ F_{w,d} \text{ (ULS)} = 17.6 \text{ kN} \quad F_{w,d} \text{ (SLS)} = 13.0 \text{ kN} \quad F_{w,d} \text{ (1 year)} = 7.94 \text{ kN} \]

Use the above EN 1991 forces in sections 2.4 & 2.5

---

**Example 2 Section 4: Support design (for either method)**

### 2.4.1 Ultimate Design Action

\[ F_{w,d} \text{ (ULS)} = 17.6 \text{ kN} \]

### 2.4.2 Ultimate Action Effects

Ultimate design bending moment per post, $M_d$

\[ M_d = \text{Wind force } \times \text{ lever arm to foundation } / \text{ number of posts} \]
\[ = F_{w,d} \text{ (ULS)} \times (z+h_b) / n \]
\[ = 17.6 \times (2.75 + 0.075) / 2 = 24.86 \text{ kNm} \]

Ultimate design shear per post, $V_d = \text{Wind force } / \text{ number of posts}$
\[ = F_{w,d} \text{ (ULS)} / n \]
\[ = 17.6 / 2 = 8.8 \text{ kN} \]

The 0.5 kN point load on the sign is not critical, since it is less than the wind action and there are no torsional effects with 2 posts.
2.4.3 Support Properties

Circular Hollow Section, CHS 168.3 × 5.0 (S355) is to be tried

Characteristic member capacities

\[ M_c = 43.5 \text{ kNm} \]
\[ P_v = 328.0 \text{ kN} \]

2.4.4 Partial Material Factor for Sign Support

\[ \gamma_m = 1.05 \text{ for steel sections (elongation > 15%)} \]

2.4.5 Ultimate Capacity Check

Ultimate bending capacity = \( M_c / \gamma_m = 43.5 / 1.05 = 41.43 \text{ kNm} \)

41.43 kNm > 24.86 kNm \text{ OK} \)

Ultimate shear capacity = \( P_v / \gamma_m = 328 / 1.05 = 312.38 \text{ kN} \)

312.38 kN > 8.80 kN \text{ OK} \)

2.4.6 Temporary Deflection Calculations

\( F_{w,d \ (1 \text{ year})} = 7.94 \text{ kN} \) \text{ section 2.3.5 above} \)

Uniformly distributed load along sign face = \( F_{w,d \ (1 \text{ year})} / b \)

\[ F_{w,d \ (1 \text{ year})} / b = 7.94 / 2.5 = 3.18 \text{ kN/m} \]

where \( b = \text{height of the sign face} \)

Maximum deflection at top of sign (bending), \( \delta \)

\[ \delta = \frac{F_{w,d\ (1 \text{ year})} / b \left( 3(H + h_b)^4 - 4(h_m + h_b)^3(H + h_b) + (h_m + h_b)^4 \right)}{24EI \cdot n} \]

where

- \( E = \text{Elastic modulus of structural steel, taken as } 205 \times 10^3 \text{ N/mm}^2 \)
- \( I = \text{Moment of inertia} \)
- \( I = 856 \text{ cm}^4 \) (for CHS 168.3 × 5.0) \( = 856 \times 10^4 \text{ mm}^4 \)
- \( n = \text{number of posts} \)
- \( H, h_b \text{ and } h_m \text{ as illustrated on diagram} \)
\[ \delta = \frac{3.18}{24 \times 205 \times 10^3 \times 856 \times 10^3 \times 2} \times \left[ 3 \times (4000 + 75)^4 - 4 \times (1500 + 75)^3 \times (4000 + 75) + (1500 + 75)^4 \right] \]

\[ \delta = 29.06 \text{ mm} \]

Deflection per linear metre, \( \delta' = \frac{\delta}{(H + h_b)} = \frac{29.06}{(4.0 + 0.075)} = 7.13 \text{ mm/m} \)

Maximum temporary deflection taken as class TDB4 = 25 mm/m

25 mm/m > 7.13 mm/m \( \text{OK} \)

2.4.7 Conclusion

2 no. CHS 168.3 × 5.0 (S355) supports are sufficient for the design.

A smaller section might also be suitable.

Notes:

Design forces may also include the moment and shear force from the wind action on the posts.

Torsion has not been calculated. This should be considered for signs that are fixed eccentrically on the posts, or where a significant area could be shielded from the wind or where buffeting from traffic can occur.

The SLS deflection limits may be more onerous for signs with moving parts

---

Example 2 Section 5: Foundation Design to BS EN 1992 & BS EN 1997

2.5.1 Characteristic Design Action

\[ F_{w,d} = 13.0 \text{ kN} \]

To provide compatibility with BS EN 1997-1, this action is referred to as \( F_{\text{rep}} \) in the remainder of the calculation.

2.5.2 Assumptions

Soil Parameters are taken to be as follows:

<table>
<thead>
<tr>
<th>Parameter ((X_u))</th>
<th>M1</th>
<th>M2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factor</td>
<td>Value ((X_d)) Factor</td>
<td>Value ((X_d))</td>
</tr>
<tr>
<td>Undrained Shear Strength, (c_u) (kPa)</td>
<td>29.00 (\gamma_{cu} = 1.0)</td>
<td>29.00 (\gamma_{cu} = 1.4)</td>
</tr>
</tbody>
</table>

Partial factors applied to \(c_u\) are those for the GEO and STR limit states. Those for the EQU limit state are significantly different.

Unit weight of soil, \(g_{\text{soil}}\) taken as 19 kN/m³
2.5.3 Foundation dimensions

Try foundation dimensions: $W = 3.40 \text{ m}$, $L = 2.10 \text{ m}$, $T = 1.50 \text{ m}$

Note: most efficient design will minimise $W$ and increase $L$ until it passes.

Unit weight of reinforced concrete $g_{\text{con}}$ taken as 24 kN/m$^3$

Characteristic weight of the foundation

$$W_k = 3.40 \times 2.10 \times 1.50 \times 24 = 257 \text{ kN}$$

2.5.4 Geotechnical Limit States for Consideration

BS EN 1997-1:2004 clause. 6.2 requires that an appropriate list is compiled from the following limit states for the design of spread foundations:

1. loss of overall stability;
2. bearing resistance failure, punching failure, squeezing;
3. failure by sliding;
4. combined failure in the ground and in the structure;
5. structural failure due to foundation movement;
6. excessive settlements;
7. excessive heave due to swelling, frost and other causes;
8. unacceptable vibrations.
Given the low complexity and the low geotechnical risk associated with the design, Geotechnical Category 1 is considered appropriate as defined in BS EN 1997-1 Cl. 2.1(14) to 2.1(16). Where this is not considered appropriate, more rigorous assessment would be necessary. Given the negligible risk of significant ground movements, limit states 5, 7 and 8 and not considered to require further consideration.

Limit states 1, 2, 3 and 6 are considered in this section; as ground movements are expected to be insignificant and any soil-structure interaction is expected to be simple in nature, limit state 4 will be considered through separate consideration of the GEO and STR limit states.

### 2.5.5 EQU Limit State

*Loss of equilibrium of the structure or the ground, considered as a rigid body, in which the strengths of structural materials and the ground are insignificant in providing resistance*

In geotechnical design, *EQU* limit state is unlikely to be critical except in rare cases, such as a rigid foundation bearing on rock and is, in principle, distinct from overall stability or buoyancy problems

\[
E_{\text{dst,d}} \leq E_{\text{stb,d}} + T_d 
\]

where:

\[
E_{\text{dst,d}} = \text{Sum of design values of the effects of destabilising actions} 
\]

\[
E_{\text{stb,d}} = \text{Sum of design values of the effects of stabilising actions} 
\]

\[
T_d = \text{Design Value of shearing resistance that develops around or on the part of the structure in contact with the ground} 
\]

**Destabilising Moment about A, \(E_{\text{dst,k}}\): Variable Unfavourable**

\[
E_{\text{dst,k}} = F_{\text{rep}} \cdot (z + h_b + T) 
\]

\[
= 13.0 \times (2.75 + 0.075 + 1.50) 
\]

\[
= 56.23 \text{ kNm} 
\]

\[
\gamma_{\text{G,dst}} = 1.50 
\]

\[
E_{\text{dst,d}} = E_{\text{dst,k}} \cdot \gamma_{\text{G,dst}} 
\]

\[
= 56.23 \times 1.50 
\]

\[
= 84.34 \text{ kNm} 
\]

**Restoring Moment about A, \(E_{\text{stb,d}}\): Permanent Favourable**

Ignoring \(Y\), as only concerned with rotational stability in this state.

\[
E_{\text{stb,k}} = W_k \cdot (L/2) 
\]

\[
= 257 \times (2.10 / 2) 
\]

\[
= 270 \text{ kNm} 
\]

\[
\gamma_{\text{G,stb}} = 0.90 
\]

\[
E_{\text{stb,d}} = E_{\text{stb,k}} \cdot \gamma_{\text{G,stb}} 
\]

\[
= 270 \times 0.90 
\]

\[
= 243 \text{ kNm} 
\]

**Shearing Resistance, \(T_d\)**

\[
T_d = 0 
\]

From the relatively low horizontal load imposed by the wind, shearing resistance may be sensibly discounted in this case, so we take the moment about A due to shearing to be zero.
EQU Limit State check
\[ E_{\text{est.d}} < E_{\text{stb.d}} + T_d \]
\[ 84.34 < 243 + 0 \quad \text{OK} \]

2.5.6 GEO Limit State

Failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance

Overturning

This check is to ensure local bearing pressure failure causing rotation of the foundation cannot occur. The analysis may be omitted on clay soils for transient wind loading and if it is considered an acceptable risk that the foundation might tilt in an extreme wind event.

Internal failure or excessive deformation of the structure or structural elements, including e.g. footings, piles or basement walls, in which the strength of structural materials is significant in providing resistance.

2.5.7 Design Approach 1  Design Combination 1: A1 + M1 + R1

where "+" implies: to be combined with

\[ E_d \leq R_d \]

where:
\[ E_d = \text{Sum of design values of the effects of actions} \]
\[ R_d = \text{Sum of design values of the resistances} \]

Destabilising Moment about A, \( E_d \): Variable Unfavourable

\[ E_k = F_{\text{rep}} (z + h_b + T) \]
\[ = 13.0 \times (2.75 + 0.075 + 1.50) \]
\[ = 56.23 \text{ kNm} \]
\[ \gamma_F = 1.50 \]
\[ E_d = E_k \cdot \gamma_F \]
\[ = 56.23 \times 1.50 \]
\[ = 84.34 \text{ kNm} \]

Restoring Moment about A, \( R_d \): Permanent Favourable

\[ R_k = W_k (L/2) - Y_k \times x \]

Resolving vertically:
\[ Y_k = W_k \]
\[ = 257 \text{ kN} \]
\[ R_k = 257 \times (2.10 / 2) - 257x \]
\[ = 270 - 257x \text{ kNm} \]
\[ \gamma_F = 1.00 \]
\[ R_d = (R_k \cdot \gamma_F) / \gamma_R \]
\[ \gamma_R = 1.00 \]
\[ R_d = [(270 - 257x) \times 1.00] / 1.00 \]
\[ = 270 - 257x \text{ kNm} \]
To achieve requirements for stability at this limit state:

\[ E_d = R_d \]

84.34 = 270 – 257\( x \)

257\( x \) = 270 – 84.34

\[ x = \frac{(270 – 84.34)}{257} = 0.72 \text{ m} \]  
(Combination 1)

\[ E_d \leq R_d \]

subject to assessment of bearing capacity and settlement check.

**2.5.8 Design Approach 1  Design Combination 2: A2 + M2 + R1**

where "+" implies: to be combined with

**Destabilising Moment about A, \( E_d \): Variable Unfavourable**

\[ E_k = F_{rep} (z + h_b + T) \]

= 13.0 \times (2.75 \div 0.075 \div 1.50)

= 56.23 \text{ kNm}

\[ \gamma_F = 1.30 \]

\[ E_d = E_k \cdot \gamma_F \]

= 56.23 \times 1.30

= 73.09 \text{ kNm}

**Restoring Moment about A, \( R_d \): Permanent Favourable**

\[ R_k = W_k \times (L/2) – Y \cdot x \]

Resolving vertically:

\[ Y_k = W_k \]

= 257 \text{ kN}

\[ R_k = 257 \times (2.10/2) – 257x \]

= 270 – 257\( x \) \text{ kNm}

\[ \gamma_F = 1.00 \]

\[ R_d = (R_k \cdot \gamma_F) / \gamma_R \]

\[ \gamma_R = 1.00 \]

\[ R_d = [(270 – 257x) \times 1.00] \div 1.00 \]

= 270 – 257\( x \) \text{ kNm}

To achieve requirements for stability at this limit state:

\[ E_d = R_d \]

73.09 = 270 – 257\( x \)

257\( x \) = 270 – 73.09

\[ x = \frac{(270 – 73.09)}{257} = 0.77 \text{ m} \]  
(Combination 2)

\[ E_d \leq R_d \]

subject to assessment of bearing capacity and settlement check.
2.5.9  Base Sliding

**Design Approach 1  Design Combination 1: A1 + M1 + R1**

where "+" implies: to be combined with

**Sliding Force, \( H_d \): Variable Unfavourable**

\[
\begin{align*}
H_k &= 13.00 \text{ kN} \\
\gamma_F &= 1.50 \\
H_d &= H_k \cdot \gamma_F \\
&= 13.00 \times 1.50 \\
&= 19.50 \text{ kN}
\end{align*}
\]

**Resistance Force, \( R_d \): Permanent Favourable**

\[
\begin{align*}
R_d &= A \cdot c_{ud} \\
&= (3.40 \times 2.10) \times 29 \\
&= 207.06 \text{ kN}
\end{align*}
\]

\[
H_d \leq R_d
\]

\[
19.50 \text{ kN} \leq 207.06 \text{ kN} \quad OK
\]

In accordance with BS EN 1997-1 clause 6.5.3(12), the following check must be made where air or water may come into contact with the interface between the foundation and an undrained subgrade:

\[
\begin{align*}
R_d \leq 0.4 \cdot V_d \\
V_d &= \gamma_G \cdot V_k \\
&= 1.00 \times W_k \\
&= 1.00 \times 257 \\
&= 257 \text{ kN}
\end{align*}
\]

\[
0.4 \cdot V_d = 0.4 \times 257 = 103 \text{ kN}
\]

\[
207.06 \text{ kN} > 103 \text{ kN}
\]

however 19.50 kN \( \leq \) 103 kN  \( OK \)

2.5.10  Design Approach 1  Design Combination 2: A2 + M2 + R1

where "+" implies: to be combined with

**Sliding Force, \( H_d \): Variable Unfavourable**

\[
\begin{align*}
H_k &= 13.00 \text{ kN} \\
\gamma_F &= 1.30 \\
H_d &= H_k \cdot \gamma_F \\
&= 13.00 \times 1.30 \\
&= 16.90 \text{ kN}
\end{align*}
\]

**Resistance Force, \( R_d \): Permanent Favourable**

\[
\begin{align*}
R_d &= A \cdot c_{ud} \\
&= (3.40 \times 2.10) \times 20.75 \\
&= 148.16 \text{ kN}
\end{align*}
\]

\[
H_d \leq R_d
\]

\[
16.90 \text{ kN} \leq 148.16 \text{ kN} \quad OK
\]
In accordance with BS EN 1997-1 clause 6.5.3(12), the following check must be made where air or water may come into contact with the interface between the foundation and an undrained subgrade:

\[ R_d \leq 0.4 \ V_d \]

\[ \ V_d = \gamma_c \cdot V_k = 1.00 \times W_k = 1.00 \times 257 = 257 \text{ kN} \]

\[ 0.4 \ V_d = 0.4 \times 257 = 103 \text{ kN} \]

\[ 148.16 \text{ kN} > 103 \text{ kN} \]

however 16.90 kN \leq 103 kN \quad OK

### 2.5.11 Bearing

\[ V_d \leq R_{d,v} \]

where:

\[ V_d = \text{Sum of the design values of the effects of vertical actions.} \]

\[ R_{d,v} = \text{Bearing capacity of the ground into which the foundation is constructed.} \]

#### Design Approach 1 Design Combination 1: A1 + M1 + R1

where "+" implies: to be combined with \quad EN 1997 2.4.7.3.4.2

From overturning calculation, \quad Eqn 2.5

\[ x = 0.72 \text{ m} \]

Calculate eccentricity of base reaction

\[ e = L/2 - x \]

\[ = 2.10/2 - 0.72 \]

\[ = 0.33 \text{ m} \]

\[ L/6 = 2.10/6 \]

\[ = 0.35 \text{ m} \]

0.33 < 0.35 therefore reaction is inside of middle third

**Maximum Bearing Pressure, \( V_d \)**

\[ V_d = \frac{W_k \cdot \gamma_f \cdot (1 + 6(e/L))}{WL} \]

\[ = \frac{257 \times 1.35 \times (1 + 6(0.33/2.10))}{3.40 \times 2.10} \]

\[ = \frac{346.95(1 + 6 \times 0.16)}{7.14} \]

\[ = \frac{680}{7.14} \]

\[ = 95.24 \text{ kPa} \]

**Bearing Capacity, \( R_{d,v} \)**

Where geotechnical information is not available \( R_{d,v} \) can be taken as 100 kPa

\[ R_{d,v} = c_u N_c + p_o \]

where \( c_{ud} \) for M1 = 29 kPa

\[ p_o = (h_b + T) \cdot g_{soil} = (0.075 + 1.5) \times 19 \]

\[ = 30 \text{ kPa} \]


2.5.12 Design Approach 1

From overturning calculation,
\[ x = 0.77 \text{ m} \]
Calculate eccentricity of base reaction:
\[ e = \frac{L}{2} - x \]
\[ = \frac{2.10}{2} - 0.77 = 0.28 \text{ m} \]
\[ L / 6 = \frac{2.10}{6} = 0.35 \text{ m} \]
\[ 0.28 < 0.35 \] therefore reaction is inside of middle third

Maximum Bearing Pressure, \( V_d \)
\[ V_d = \frac{W L \gamma_f (1 + 6(e/L))}{257 \times 1.00 \times (1 + 6(0.28/2.10))} \]
\[ = \frac{3.40 \times 2.10}{7.14} \]
\[ = \frac{458}{7.14} \]
\[ = 64.15 \text{ kPa} \]

Bearing Capacity, \( R_{dv} \)
Where geotechnical information is not available \( R_{dv} \) can be taken as 100 kPa
\[ R_{dv} = c_u N_c + p_o \]
Where \( c_{ua} \) for M2 = 20.75 kPa
\[ p_o = 30 \text{ kPa} \] as previously (section 2.5.11)
\[ N_C = 6.25 \] as previously (section 2.5.11)
\[ R_{dv} = 20.75 \times 6.25 + 30 = 159.69 \text{ kPa} \]
\[ V_d \leq R_{dv} \]
\[ 64.15 \text{ kPa} \leq 159.69 \text{ kPa} \] OK
### 2.5.13 Settlement Check

Check Maximum Bearing Pressure against Allowable Bearing Pressure, both calculated using characteristic values.

This check is required where any settlement is unacceptable.

#### Destabilising Moment about A, $E_k$ : Characteristic Values

- \[ E_k = F_{rep} \cdot (z + h_b + T) \]
- \[ = 13.0 \times (2.75 + 0.075 + 1.50) \]
- \[ = 56.23 \text{ kNm} \]

#### Restoring Moment about A, $R_k$ : Characteristic Values

- \[ R_k = W_k \cdot (L / 2) - Y \cdot x \]

**Resolving vertically:**

- \[ Y_k = W_k = 257 \text{ kN} \]
- \[ R_k = 257 \times (2.10 / 2) - 257x \]
- \[ = 270 - 257x \text{ kNm} \]

To achieve requirements for stability at this limit state:

\[ E_k = R_k \]

\[ 56.23 = 270 - 257x \]

\[ 257x = 270 - 56.23 \]

\[ x = (270 - 56.23) / 257 \]

\[ = 0.83 \text{ m} \]

Calculate eccentricity of base reaction:

- \[ e = L/2 - x \]
- \[ = 2.10 / 2 - 0.83 \]
- \[ = 0.22 \text{ m} \]

\[ L/6 = 2.10 / 6 \]

\[ = 0.35 \text{ m} \]

0.22 > 0.35  therefore reaction is inside of middle third

#### Maximum Bearing Pressure, $V_k$

- \[ V_k = \frac{W_k \left(1 + 6(e/L)\right)}{BL} \]
- \[ = \frac{257 \times \left(1 + 6 \times (0.22/2.10)\right)}{3.40 \times 2.10} \]
- \[ = \frac{257 \times (1 + 6 \times 0.105)}{7.14} \]
- \[ = \frac{419}{7.14} \]
- \[ = 58.6 \text{ kPa} \]

If reaction occurs outside of middle third, the following formula will be applicable:

- \[ V_k = -\frac{4W_k}{3B(L - 2e)} \]
**Ultimate Bearing Capacity, $R_{kv}$**

Where geotechnical information is not available $R_{kv}$ can be taken as 100 kPa

$$R_{kv} = c_u N_c + p_o$$

Where $c_u = 29$ kPa

$$p_o = (h_b + T)y$$

$$= 30$$

$N_c = 6.25$ as previously (section 2.5.11)

$$R_{kv} = 211.25 \text{ kPa}$$

**Allowable Bearing Pressure**

To avoid the need for detailed settlement calculations, it is assumed that settlements will be limited to suitable values if the maximum characteristic applied bearing pressure does not exceed one third of the ultimate characteristic bearing capacity.

Therefore the allowable bearing pressure:

$$R_{all,v} = 211.25 / 3 = 70.42 \text{ kPa}$$

A detailed calculation should be carried out for very weak highly compressible soils.

$$V_k \leq R_{all,v}$$

$$58.6 \text{ kPa} \leq 70.42 \text{ kPa} \quad \text{OK}$$

**2.5.14 Conclusion**

A foundation with $W = 3.40 \text{ m}$, $L = 2.10 \text{ m}$ and $T = 1.50 \text{ m}$ is sufficient for the design.

A smaller foundation, particularly one with a reduced thickness, $T$, might also be suitable.

A spread foundation should be reinforced unless it is shown that the bending and shrinkage effects are negligible. In general foundations for smaller CHS posts (88.9 mm diameter or less) do not need reinforcement.

Refer to Example 1 for planted bases, which are often an economical alternative.
APPENDIX D: REFERENCES

10. BS EN 40-2:2004 Lighting columns - General requirements and dimensions.
11. BS EN 40-3-3:2003 Lighting columns - Verification by calculation.
12. PD 6547:2004 Guidance on the use of BS EN 40-3-1 and BS EN 40-3-3, BSI.
16. TD 19/06 Requirement for Road Restraint Systems (vol 2, sect 2, part 8 of Design Manual for Roads and Bridges), Highways Agency.
17. BD 94/07 Design of Minor Structures (vol 2, sect 2, part 1 of Design Manual for Roads and Bridges), Highways Agency.
18. Free software for determining both the appropriate wind action and suitable steel support sections using the methods described in this booklet (SignLoad Designer) is available from Buchanan Computing: www.BuchananComputing.co.uk
19. Passive Safety UK Guidelines, lists of available passively safe products and other background information: www.passivesafetyuk.com
20. Institute of Highway Engineers traffic sign knowledge network: theihe.org/knowledge-network/traffic-sign-design
Disclaimer:
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